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EVOLUTION OF ACCURACY CRITERIA CONCERNING BOUNDARY POINT LOCATION IN THE CADASTRAL DATA BASE IN POLAND

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SUMMARY
The paper presents historical and contemporary technologies and accuracy requirements, related to surveys of points and boundary lines, included in the cadastral data base in Poland. The author presents the analysis of accuracy criteria within the last hundred years – beginning at the interwar period, with consideration of cadastral technical instructions, which were obligatory in areas of former partitions of Poland; the analysis continues through the times of the Polish People's Republic until the present times. The paper presents technology of restoration of initial locations of boundary points, with consideration of permitted errors, which characterised surveys performed in the past. Restoration of location of a boundary point, directly in the co-ordinate system, currently obligatory in Poland, requires consideration of errors of adjustment of former control networks, as well as errors, which were permitted for surveys of boundary points, basing on the initial control network.

1. TECHNOLOGIES OF SURVEYS AND ACCURACY REQUIREMENTS IN THE INTER-WAR PERIOD

The following technical regulations were obligatory in the interwar Poland:
- The Temporary Technical Instruction of 1920 [1],
- The Technical Instruction of 1926 [2] which substituted the Temporary Instruction [1] in then existing Białystok, Kielce, Lublin, Łódź, Nowogród, Polesie, Warszawa, Wołyń voivodships (provinces) and in the Vilnius District,
- The Cadastral Instruction II of 1927, which was obligatory in the area of the ex-Prussian annexation, [3],
- The Technical Instruction of 1931, obligatory in the area of the ex-Austrian annexation [4].

In the Central Poland, following the Instructions [1] and [2], the accuracy criteria, related to surveys of traverses were obligatory; they are listed in Tables 1 and 2.

Table 1. The biggest permissible values of linear discrepancies of traverses for selected length of a traverse; Instructions [1], [2]

Traverse length [m]	Permissible linear discrepancy [m]
500	0.44
1000	0.72
2000	1.20
4000	2.10
6000	2.95

Table 2. The biggest permissible angular discrepancies of traverses for selected numbers of angles of a traverse; instructions [1], [2]

Number of angles	Permissible angular discrepancies [‘]
5	8.9
10	12.6
20	17.9
40	25.3
60	31.0

Angles of circumferential traverses were measured in two positions of a telescope, using a 1' theodolite and sides were measured with a steel tape with the accuracy of 0.05 m. The circumferential traverse and related traverses are presented in Fig.1. Border points were surveyed from polygon points using the orthogonal method and sideshots under arbitrary angles and with the use of intersections.

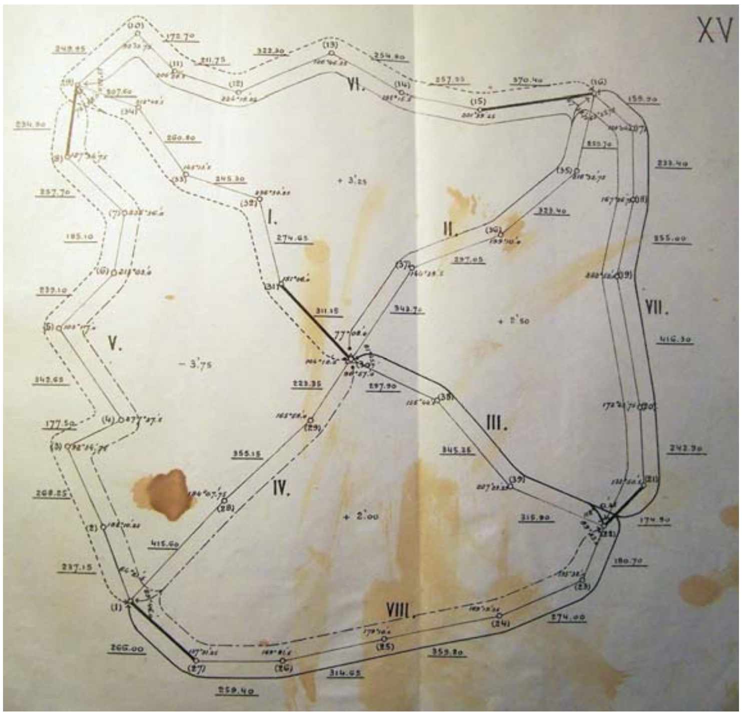

Figure 1. The circumferential traverse and related traverses; Instruction [1]

A fragment of a plan of lands of a private farm Chrzanów, Janów district, Lublin voivodship, divided into parcels, elaborated in 1930 by the sworn surveyor, Wacław Nowak, following the Instruction [2], is presented in Fig.2.

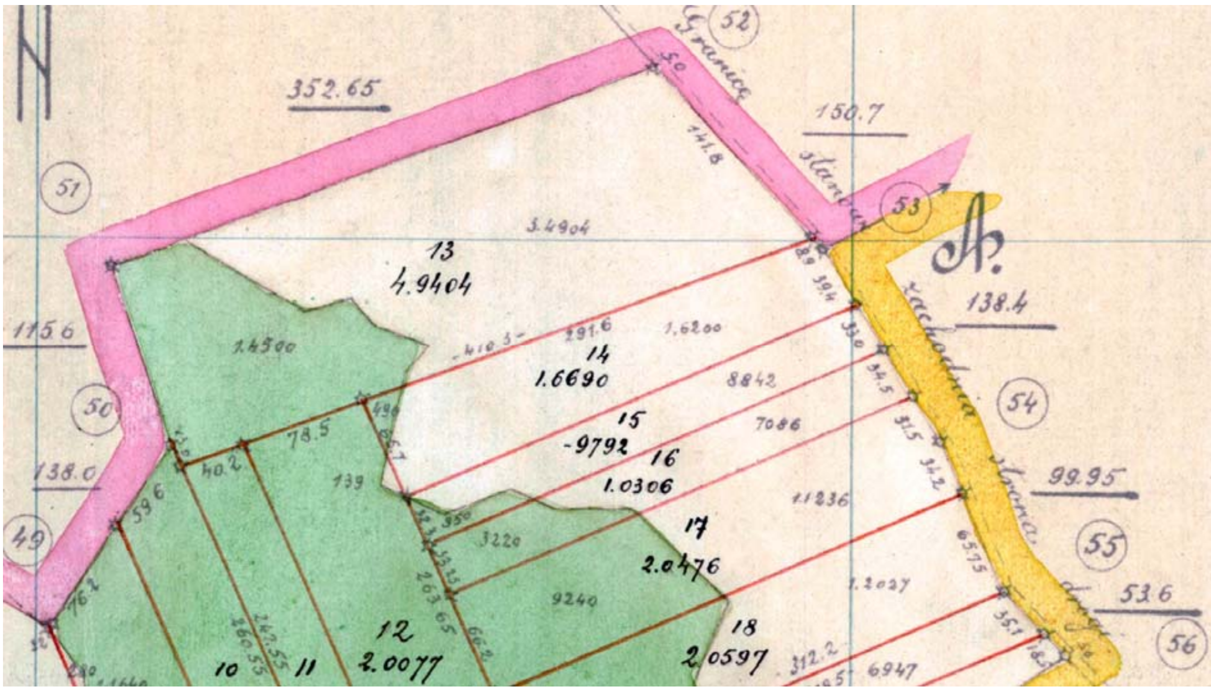

Figure 2. Fragment of the plan of lands of 1930; ZKiGN GiK PW [5]

In western voivodships the instruction [3] was obligatory, which specified the permitted differences (d) between two surveys of the length of a survey line or a traverse sequence (s), depending on the terrain features:

I – in flat plane areas: $d = 0.01\sqrt{4s - 0.005s^2}$

II – in undulating areas: $d = 0.01\sqrt{6s - 0.0075s^2}$

III – in highly diversified areas: $d = 0.01\sqrt{8s - 0.01s^2}$

For example, for a line s = 465 m the permitted difference equalled to:
- for the I areas I - 0.54 m
- for the II area II – 0.66 m
- for the III area III – 0.77 m

The instruction [4], obligatory in southern voivodships specified the limits of permitted errors Δl [cm], depending on the length l [m], for the needs of checking property boundaries basing on the cadastral map. The cadastral map, developed basing on plane table surveying, was the basis for determination of property conditions, calculation of parcel size for land consolidation purposes and for searching and checking the location of boundary points. Locations of boundary points were checked by comparing the real status with the status presented on the map and by surveys of corresponding real lengths with lengths from the cadastral map. Differences of surveys could not exceed the value calculated from the formula: $\Delta l = 2(0.00015l + 0.005\sqrt{l} + 0.015)$, providing that the value S/5000 should be added to the calculated value, where S was the denominator of the scale of the cadastral map.

2. TECHNOLOGIES OF SURVEYS AND ACCURACY REQUIREMENTS AFTER THE WORLD WAR II

The register of lands and buildings, existing on Poland, was established on the basis of regulations, which are not obligatory at present ; the decree of 1955 [6] the Technical Instruction of 1962 [7].

In order to survey boundary points, the detailed control network should be created in the surveyed are, which was connected to the basic control network. The rule was applied in the process of creation of the detailed control network, that detailed networks of lower accuracy, such as planimetric sequences, points and survey lines should be rest on detailed control networks of higher accuracy, such as intersected points, points of co-ordinates transfer and main sequences.

The limit value of the angular discrepancy of a traverse, specified in the Instruction [7] did not exceed the value calculated from the formula $f_{kt} \leq 2m_0\sqrt{n_{kt}}$, where: m0 – the accuracy of angular measurements for the given length of a sequence; nkt – the number of angles in the traverse. The difference (Δl) of the double survey of a polygon side did not exceed the value of the mean error of the survey of the particular side: $\Delta l \leq m_l$ where: $m_l = u\sqrt{l}$, and: l – the length of a polygon side; u – the coefficient of accidental errors of linear surveys.

According to Table 3, the highest permissible value of the polygon point positioning error, of the least favourable position (a central point in a traverse), related to the point of connection, equalled to 0.75 m (for the technical traverse method of the 5th class).

Table 3. Permissible errors of polygon point positioning according to the Instruction [7]

The mean point positioning error in relation to the point of connection (after adjustment)	The technical traverse method 1st class – $m_p \leq 7.5$ cm 2nd class – $m_p \leq 15$ cm 3rd class – $m_p \leq 25$ cm 4th class – $m_p \leq 50$ cm 5th class – $m_p \leq 75$ cm

Surveys of property boundaries were performed using the offset method, elongations, crosswise sides and angular and linear intersections, basing on the detailed control network. Measures were readout with the accuracy of 0.10 m.

The next technical instruction, which aggravated the accuracy criteria of surveys of horizontal networks, was the instruction [8] according to this instruction the biggest permissible point positioning error equalled to 50 cm (after adjustment), for the survey network in rural lands. Permissible point positioning errors related to the point of connection, specified by the instruction [8], are listed in Table 4.

Table 4. Permissible errors of network according to the instruction [8]

The mean point positioning error in relation to the point of connection (after adjustment)	Horizontal network 1st class – $m_D < 5 \cdot 10^{-6}$ 2nd class – $m_p \leq 5$ cm 3rd class – $m_p \leq 10$ cm Survey network – $m_p \leq 20$ cm ($m_p \leq 50$ cm for rual areas)

3. RECENT TECHNOLOGIES OF SURVEYS AND ACCURACY REQUIREMENTS

Following the Decree [9], which is obligatory in Poland, surveys of details are performed using the following methods:

1) the method of polar co-ordinates;
2) the offset method;
3) the method of angular, linear and angular-and-linear intersections;
4) the precise positioning method using the GNSS.

The recent accuracy requirements concerning horizontal networks are specified in the Decree [9] – with respect to a survey network, and the Decree [10] – with respect to networks of the 1st, 2nd and 3rd classes. Surveys of details are performed basing on points of a horizontal network; in the case when the density of points is not sufficient for performed surveys, the network is amended with points of a survey network. Mean point positioning errors of the network, related to the point of connection of the horizontal network of the 1st – 3rd classes and of the survey network are presented in Table 5.

Table 5. Mean points positioning errors of the network, following the Decrees [9], [10]

The mean point positioning error in relation to the point of connection (after adjustment)	Horizontal network 1st class - $m_p \leq 1$ cm 2nd class $m_p \leq 5$ cm 3rd class $m_p \leq 10$ cm Survey network: $m_p \leq 10$ cm

4. TECHNOLOGICAL ASPECTS OF DETERMINATION OF SURVEY POINT POSITIONS

Following the Decree [9] surveys of details, aiming at re-establishing of boundary marks or determination of boundary points, are performed with the use of observation data, which specified the position of those marks or boundary points, basing on the survey network, which was used for acquisition of this data (the primary network). In the case when the primary network does not exist (due to damages or displacements of its points, or due to impossibility of reestablishment of this network), surveys of details aiming ar reestablishment of boundary marks or determination of boundary points are performed basing on topographic descriptions of these boundary points or co-ordinates of these boundary points, after harmonisation of these points, using the mathematical transformation, with the reference system, specified by points of the horizontal network and the survey network.

Reestablishment of boundary points or determination of boundary points may be performed with the use of the global system of satellite navigation, „GNSS", using the static, fast static or kinematic RTK or RTN techniques. As it turns out from the expertise commissioned by the Head Office of Geodesy and Cartography (GUGiK) [11], the basic of unification of surveying and cartographic works in Poland is ensured by the ASG-EUPOS system, operating in Poland since 2008, which guarantees the correctness of implementation of the state system of spatial references.

The ASG-EUPOS system is used for surveys with the precise positioning method, with the use of GNSS. Prior to or in the course of each session of surveys with the use of RTK and RTN kinematic techniques, the control measurement is performed for at least two points of the horizontal control network, located within the distance not longer that 5 km from the points, which are the subject of surveys. The linear discrepancy, specified basing on control measurements, cannot exceed 0.12 m with respect to plane rectangular co-ordinates.

Following Ryczywolski [12], the RTK method of survey, performed by the NAWGEO service of the ASG-EUPOS service, is characterised by the positioning precision of 0.03 – 0.05 m, with respect to reference stations, which have the co-ordinates in the obligatory reference system (ETRS). At the beginning of RTK surveys, the mobile receiver performs the, so-called, initialisation, when its determines its precise position. After initialisation it is ready to operate. The probability that the GNSS receiver performs incorrect initialisation is very low. As a result of erroneous initialisation, displacement of all surveyed points by the constant vector may occur; the length of that vector is the multiplication of the wavelength in the satellite system (about 20 cm). Due to that purpose surveys should be started at the control point of known co-ordinates.

Following the interpretation of GUGiK [13], operations concerning the reestablishment of a boundary mark or determination of a boundary point, aim at restoration of the position of the particular mark or the point on the ground, basing on the documentation, which specifies its primary position, i.e. basing on the same points of the horizontal network, which was used for the needs of initial surveys, as well as with the use of observation data, including control data.

Only in the case of lack of the possibility to restore the geodetic network, which was the basis for original surveys, the existing network may be used for determination of boundary points, following appropriate operations, aiming at optimization of the accuracy of co-ordinates of determined restored boundary marks or determined boundary points, with respect to the current network. They are the following operations:
1) geodetic surveys which allow for readjustment of the primary network in connection to the current basic or detailed network, and then for recalculation of co-ordinates of determined points,
2) transformation of co-ordinates of determined points, basing on the sufficient number of tie points, which co-ordinates are calculated basing on original surveys, as well as on surveys performed basing on the current network; the co-ordinate system of the original surveys is considered as the primary system and the co-ordinate system where the co-ordinates of the current network are determined, is considered as the secondary system.

In the ASG-EUPOS system surveys are performed by reference stations, which are directly connected with the basic geodetic network; therefore the additional connection to the network terrestrial points is not necessary. However, this relation is an important issue connected with determination of positions of boundary points, which were not initially directly surveyed basing on the basic network, but they were surveyed basin on lower order networks or networks in local systems. Boundary points positioning errors with errors referring to networks, being the basis for surveys of boundary points and errors of boundary points, directly determined from the basic ASG-EUPOS network, are presented in Table 6. Boundary pints positioning errors with reference to the basic network presented in the fifth column of Table 6 have been calculated with consideration of accuracy criteria listed in Tables 3-5, with the use of the general formula for calculation of the mean function error, implemented as the square root of the total of squares of errors of particular classes of lower order networks.

Table 6. Point positioning errors for networks and boundary points (elaborated by the author)

Type of a network	**Point positioning error with respect to the survey network [m]**	**Point positioning error of the survey network with respect to the detailed network [m]**	**Point positioning error of the detailed network with respect to the basic network [m]**	**Point positioning error of a boundary point with respect to the basic network [m]**
Technical traversing, instruction [7], 1962	0.20	0.75	0.50	**0.93**
Horizontal network, instruction [8], 1988 r.	0.10	0.20 (city) 0.50 (village)	0.10	**0.25** **0.52**
Horizontal network, Decree [9], 2011 r.	0.10	0.10	0.10	**0.17**
ASG EUPOS	-	-	-	**0.05**

As it turns out from Table 6, the boundary point positioning error – surveyed from the survey network, adjusted to the detailed network, which was adjusted to the basic network (with the permissible mean error), is close to 1 metre. The direct survey from the current basic ASG-EUPOS network – being theoretically almost free of errors, results in neglecting errors of adjustment of primary network, and in practice, the boundary point positioning error equal to about 1 metre. The boundary point positioning error with respect to the basic network froma Table 6 has been illustrated in Diagram 1 (colours from the table correspond to colours from the diagram). The vertical axis presents the error in metres, the horizontal axis presents three accuracy periods: 1 – 1962, 2 – 1988, and 3 – 2012.

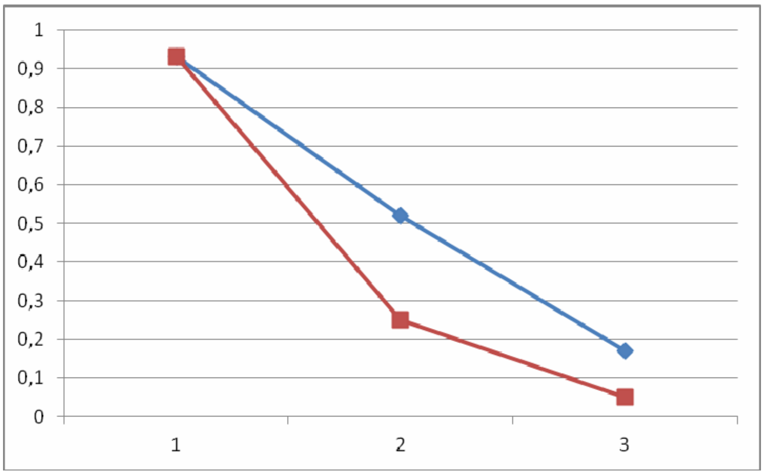

Diagram 1. Evolution of accuracy criteria (elaborated by the author)

5. AN EXAMPLE OF OPTIMISATION OF THE ACCURACY OF CO-ORDINATES OF DETERMINED BORDER POINTS

The example of optimisation of accuracy of co-ordinates of determined boundary points is to practically illustrate the impact of gradual loss of initial data from surveys of the network on the accuracy of determination of points of this network.

Figure 3 presents the location of 13 points of the network. All error-free data concerning this network – co-ordinates of points, angles and distances between points, are known (Table 7).

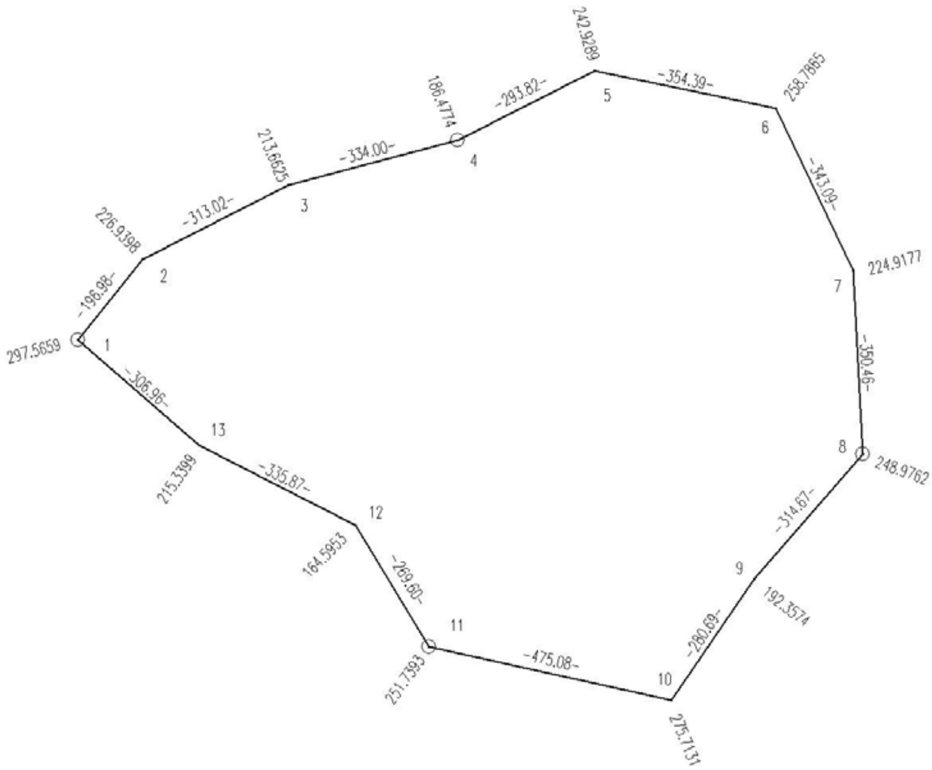

Figure 3.

Table 7.

			u=	0,008		m0=	0,0167
Współrzędne prawdziwe							
Nr	X	Y	Długość boku	Max. błąd długości [m]		Kąt lewy	Max. błąd kąta [g]
1	1000,00	1000,00					
2	1153,70	1123,20	196,98	0,11		226,9398	0,0167
3	1296,00	1402,00	313,02	0,14		213,6625	0,0167
4	1381,00	1725,00	334,00	0,15		186,4774	0,0167
5	1514,00	1987,00	293,82	0,14		242,9289	0,0167
6	1442,00	2334,00	354,39	0,15		258,7865	0,0167
7	1132,00	2481,00	343,09	0,15		224,9177	0,0167
8	782,00	2499,00	350,46	0,15		248,9762	0,0167
9	545,00	2292,00	314,67	0,14		192,3574	0,0167
10	313,00	2134,00	280,69	0,13		275,7131	0,0167
11	415,00	1670,00	475,08	0,17		251,7393	0,0167
12	646,00	1531,00	269,60	0,13		164,5953	0,0167
13	799,00	1232,00	335,87	0,15		215,3399	0,0167
1	1000,00	1000,00	306,96	0,14		297,5659	0,0167
			Max. odchyłka liniowa [m]:	0,52		Max. odchyłka kątowa [g]:	0,1204

The first variant of optimisation ia based on the assumption that some geodetic marks of the network points were damaged and co-ordinates of those points are unknown. This variant implements the guidelines [13] by the readjustment of the primary network in connection to the current basic or detailed network, and then by calculation of co-ordinated of determined points. In order to determine co-ordinates of missing points, known co-ordinates of existing points of the traverse and complete survey data for the traverse are applied. It has been assumed in the first variant that points: 3p, 6p, 7p, 10p, 13p were damaged. In order to calculate co-ordinates of those points, calculations were performed for four traverses, angularly and linearly tied us at both ends. Results of calculations performed using the C-Geo software tool, are presented in Tables 8 – 11.

Table 8

CIĄG DWUSTRONNIE NAWIĄZANY

Punkty nawiązania :

Numer	**X**	**Y**	**Azymut**
1p	1000.05	999.95	
2p	1153.75	1123.15	
4p	1381.05	1724.95	
5p	1514.05	1986.95	

Dane ciągu

Numer	**Kąt**	**Bok**	**X**	**Y**
2p	226.9560	313.10	1153.75	1123.15
3p	213.6800	333.90	1296.11	1402.03
4p	186.4650		1381.05	1724.95

Długość ciągu : 647.00

Odchyłki ciągu:

fk = 0.0212, fl = 0.128,

fk max = 0.0312, fl max = 0.192,

fx = 0.125, fy = -0.028,

Table 9

CIĄG DWUSTRONNIE NAWIĄZANY

Punkty nawiązania :

Numer	**X**	**Y**	**Azymut**
4p	1381.05	1724.95	
5p	1514.05	1986.95	
8p	782.05	2498.95	
9p	545.05	2291.95	

Dane ciągu

Numer	**Kąt**	**Bok**	**X**	**Y**
5p	242.9130	354.45	1514.05	1986.95
6p	258.7710	343.00	1442.00	2334.00
7p	224.9030	350.40	1132.03	2480.96
8p	248.9610		782.05	2498.95

Długość ciągu : 1047.85

Odchyłki ciągu:

fk = -0.0614, fl = 0.136,

fk max = 0.0360, fl max = 0.240,

fx = -0.134, fy = -0.022,

Table 10

CIĄG DWUSTRONNIE NAWIĄZANY

Punkty nawiązania :

Numer	**X**	**Y**	**Azymut**
8p	782.05	2498.95	
9p	545.05	2291.95	
11p	415.05	1669.95	
12p	646.05	1530.95	

Dane ciągu

Numer	**Kąt**	**Bok**	**X**	**Y**
9p	192.3660	280.60	545.05	2291.95
10p	275.7200	475.15	313.09	2134.00
11p	251.7440		415.05	1669.95

Długość ciągu : 755.75

Odchyłki ciągu:

fk = 0.0202, fl = 0.113,

fk max = 0.0312, fl max = 0.205,

fx = -0.111, fy = 0.021,

Table 11

CIĄG DWUSTRONNIE NAWIĄZANY

Punkty nawiązania :

Numer	X	Y	Azymut
11p	415.05	1669.95	
12p	646.05	1530.95	
1p	1000.05	999.95	
2p	1153.75	1123.15	

Dane ciągu

Numer	Kąt	Bok	X	Y
12p	164.6100	335.96	646.05	1530.95
13p	215.3500	306.90	799.09	1231.88
1p	297.5600		1000.05	999.95

Długość ciągu : 642.86

Odchyłki ciągu:

fk = 0.0189, fl = 0.088,

fk max = 0.0312, fl max = 0.191,

fx = -0.084, fy = -0.027,

Table 12 presents the true errors of co-ordinated of points 3p, 6p, 7p, 10p, 13p obtained from caqlculations, which were calculated with consideration of permissible mean errors of surveys of network points, with the assumption that the true co-ordinates of points 3p, 6p, 7p, 10p, 13p are known. As a result of performed analysis the true errors of points 3p, 6p, 7p, 10p, 13p were obtained as a result of calculations of traverses angularly and linearly tied up at both ends, which, in the worst case, were equal to 0.15 m point 13p).

Table 12

Wariant I - ciągi dwustronnie nawiązane							
Współrzędne pomierzone				Współrzędne obliczone			
Nr	X	Y	Błąd prawdziwy	Nr	X	Y	Błąd prawdziwy
1p	1000,05	999,95	0,07				
2p	1153,75	1123,15	0,07				
				3p	1296,11	1402,03	0,11
4p	1381,05	1724,95	0,07				
5p	1514,05	1986,95	0,07				
				6p	1442,00	2334,00	0,00
				7p	1132,03	2480,96	0,05
8p	782,05	2498,95	0,07				
9p	545,05	2291,95	0,07				
				10p	313,09	2134,00	0,09
11p	415,05	1669,95	0,07				
12p	646,05	1530,95	0,07				
				13p	799,09	1231,88	0,15

Implementation of the second variant, following the guidelines [13],comprised transformation of co-ordinates of points determined on the basis of the sufficient number of tie points, which co-ordinates are calculated on the basis of original surveys, as well as surveys based on the current network; in the latter case the co-ordinate system of the original survey was considered as the primary system and the co-ordinate system, in which co-ordinates of the current network are determined, was considered as the secondary system. It was assumed that points 1p, 4p, 8p, 11p were known and co-ordinates of remaining points (9 out of 13) of the traverse were calculated from transformation. Results of analysis are presented in Tables 13-14. Table 13 presents the report of C-Geo tool and Table 14 presents the obtained true errors, reaching 0.22 in the worst case (point 13t).

Table 13

TRANSFORMACJA WSPÓŁRZĘDNYCH (AFINICZNA)

Punkty dostosowania

Nr p	Xp	Yp	Nr w	Xw	Yw
1	1000.00	1000.00	1p	1000.05	999.95
4	1381.00	1725.00	4p	1381.05	1724.95
8	782.00	2499.00	8p	782.05	2498.95
11	415.00	1670.00	11p	415.05	1669.95

Punkty transformowane

Nr p	Xp	Yp	Hp	Nr w	Xw	Yw	Hw
2p	1153.75	1123.15		2t	1153.80	1123.10	
3p	1296.11	1402.03		3t	1296.16	1401.98	
5p	1514.05	1986.95		5t	1514.10	1986.90	
6p	1442.00	2334.00		6t	1442.05	2333.95	
7p	1132.03	2480.96		7t	1132.08	2480.91	
9p	545.05	2291.95		9t	545.10	2291.90	
10p	313.09	2134.00		10t	313.14	2133.95	
12p	646.05	1530.95		12t	646.10	1530.90	
13p	799.09	1231.88		13t	799.14	1231.83	

Table 14

Wariant II - transformacja współrzędnych

Współrzędne pomierzone				Współrzędne z transformacji			
Nr	X	Y	Błąd prawdziwy	Nr	X	Y	Błąd prawdziwy
1p	1000,10	999,90	0,07				
				2t	1153,80	1123,10	0,14
				3t	1296,16	1401,98	0,16
4p	1381,10	1724,90	0,07				
				5t	1514,10	1986,90	0,14
				6t	1442,05	2333,95	0,07
				7t	1132,08	2480,91	0,12
8p	782,10	2498,90	0,07				
				9t	545,10	2291,90	0,14
				10t	313,14	2133,95	0,15
11p	415,10	1669,90	0,07				
				12t	646,10	1530,90	0,14
				13t	799,14	1231,83	0,22

Table 15 presents positioning errors of such points, which co-ordinates were calculated in both variants.

Table 15

Błędy wyznaczenia położenia punktów [m]					
Współrzędne prawdziwe			Błąd - wariant I	Błąd - wariant II	Różnica
Nr	X	Y			
1	1000,00	1000,00			
2	1153,70	1123,20			
3	1296,00	1402,00	0,11	0,16	0,05
4	1381,00	1725,00			
5	1514,00	1987,00			
6	1442,00	2334,00			
7	1132,00	2481,00	0,05	0,12	0,07
8	782,00	2499,00			
9	545,00	2292,00			
10	313,00	2134,00	0,09	0,15	0,06
11	415,00	1670,00			
12	646,00	1531,00			
13	799,00	1232,00	0,15	0,22	0,07

It results from Table 15 that the readjustment of the primary network, performed in connection with the current basic or detailed network, and re-calculation of co-ordinates of determined points is more accurate determination of co-ordinates of unknown points comparing to transformation.

6. FINAL REMARKS AND CONCLUSIONS

Gradual aggravation of accuracy criteria of surveys of networks and surveys of boundary points proves that determination of the primary position of border points should be performed directly from the network, which was the basis for original surveys of those points. The direct determination of point positions in connection to the ASG-EUPOS levels errors of the networks, to which the original survey was connected. As a result the point directly determined from the basic ASG_EUPOS network, without consideration of relations of that network with the primary network, will not overlap its primary position.

As it turns out from performed optimisation of accuracy of co-ordinates of determined boundary points:

- In order to determine boundary point positions, operations reverse to the applied survey technique, which was the basis of original surveys of points, should be applied,
- The highest accuracy of determination of boundary point positions is achieved using the same network points and the same observation data, which were used form the original survey,
- Transformation of co-ordinates should be applied when there is no data for determination of co-ordinates of the original network basing on the traverse adjustment or the rigorous adjustment of the related traverse,
- In the case of necessity of application of transformation, all points of the traverse (which were originally fixed and which positions were not destroyed, displaced or damaged) should be found and surveyed within the entire district,
- Obtained boundary point positioning errors (exceeding the accuracy of topographic details of the 1st accuracy group) point that - if fixed topographic details are located within the area of the determined error – it is highly probable that they overlap the position of boundary lines.

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